AESCO
TECHNOLOGIES, INC.

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GEOTECHNICAL REPORT
PROPOSED T-MOBILE WIRELESS COMMUNICATIONS FACILITY
SITE NUMBER: SV11110D
SITE NAME: SHOREHEIGHTS DRIVE ROW
3418 ½ SHOREHEIGHTS DRIVE
SANTA MONICA, CA
AESCO PROJECT NO. 20080885-A3309

Prepared for:

T-Mobile USA
4100 Guardian Street
Simi Valley, CA 93063

Attention: Mr. Dan Rico

Prepared By:

AESCO Technologies, Inc.
17782 Georgetown Lane
Huntington Beach, California 92648

Adam Chamaa, P.E., Manager

July 31, 2008

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Mr. Dan Rico
T-Mobile USA
4100 Guardian Street
Simi Valley, CA 93063

Subject: **Geotechnical Report**
Proposed T-Mobile Wireless Communications Facility
Site Number: SV11110D
Site Name: 3418 ½ Shoreheights Drive
Shoreheights ROW
Malibu, CA
AESCO Project No. 20080885-A3309

Dear Mr. Rico:

AESCO Technologies, Incorporated (AESCO) is pleased to provide you three (3) copies of the geotechnical report for the proposed communications facility to be constructed at the subject site.

AESCO will be happy to assist you further on this project by furnishing any Construction Materials Testing and Inspection Services you may require during the construction phase of the project. We are a full service-testing laboratory and inspection service and can supply the full range of testing and inspection services such as soils, concrete, asphalt, steel, welding, etc. that may be necessary for construction of this project.

Please do not hesitate to contact us if you have any questions or if we may be of any additional assistance. We look forward to assisting you during the construction of the proposed facility.

Sincerely,
AESCO Technologies, Inc.

Debra L. Perez
Project Manager

REGISTERED PROFESSIONAL ENGINEER
ADAM CHAMAA
GE 2784
Exp. 12/31/09
GEOTECHNICAL
STATE OF CALIFORNIA

Adam Chamaa, MSCE, P.E., G.E.
Senior Project Engineer

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Geotechnical Report
Proposed T-Mobile Wireless Communications Facility
Site Name: Shoreheights Drive ROW
Site Number: SV11110D
3418 ½ Shoreheights Drive
Malibu, CA

This report (authorized by T-Mobile as defined in our proposal dated May 16, 2005), presents the results of a geotechnical investigation performed by AESCO Technologies, Incorporated (AESCO) for a proposed communications facility and support structure to be installed at 3418 12 Shoreheights Drive, Malibu, California. The layout of the proposed facility is shown on the Site Plan, Figure 1.

We understand that the communications tower will consist of an approximately 23-foot tall monopole. The equipment support structure will consist of a pre-cast underground reinforced concrete vault with outside plan dimensions of approximately 8 feet by 11 feet, and a height of approximately 17 feet. The top of the vault will be just below the ground surface. The equipment cabinets will be supported on the floor slab of the vault. The vault will be placed east of the monopole (on the opposite side of the street). Axial, base shear and overturning loads for the proposed monopole were not available at the time of this report.

The purpose of this study was to provide geotechnical input for the design of the monopole and vault. The scope of our services included the following:

- Coordinating site access for the field investigation;
- Obtaining utility clearances for the field investigation;
- Performing geotechnical drilling and sampling at the site;
- Performing laboratory testing of representative samples;
- Conducting a seismic hazards screening;
- Obtaining a permit prior to drilling;
- Preparing a traffic control plan;
- Performing traffic control;
- Engineering analyses; and
- Preparing this report.

This report summarizes our findings and presents geotechnical recommendations for the design of this communications facility.

2.1 FIELD INVESTIGATION

A field investigation was conducted at the site on July 10, 2008 to obtain information on the subsurface conditions. The field investigation consisted of drilling one hollow-stem auger boring. Boring B-1 was drilled to a depth of 40 feet below the existing ground surface, as shown on the Site Plan, Figure 1. The site plan is based on a proposed site layout drawing by BMS Communications, Inc., dated February 25, 2008. AESCO personnel logged the boring and visually classified and collected samples of the subsurface materials encountered in the boring. The boring was backfilled with cuttings. The Log of Boring B-1 is presented in the attached Appendix.

Drive samples were taken in the boring using either a Standard Penetration Test (SPT) sampler or a Modified California (MC) sampler. The sampler was driven 18 inches into the bottom of the borehole using a 140-pound hammer falling a distance of 30 inches. The MC sampler barrel was lined with stainless steel liners to collect relatively undisturbed soil samples. All of the samples were sealed and packaged to help preserve the natural moisture content and to protect them from further disturbance.

2.2 LABORATORY TESTING

All testing was performed in accordance with ASTM Standards and California Test Methods. Laboratory testing performed in our Huntington Beach, California geotechnical laboratory consisted of water content (ASTM D4959), dry density (ASTM D2937), direct shear (ASTM D3080), Atterberg Limits (ASTM D4318), and washed sieve analysis (ASTM D1140). Results of the laboratory tests are summarized on the Boring Log and are included in the attached Appendix. Chemical analyses, including pH (ASTM D1293), soluble sulfates (CT417) and soluble chlorides (CT422) were also performed. Chemical test results are presented in Section 4.8.

3.1 REGIONAL GEOLOGIC SETTING

The site is located within the Los Angeles Basin, near the northern boundary of the Peninsular Ranges Physiographic Province. The Peninsular Ranges Physiographic Province is characterized by northwest-trending topographic structures, including the Newport Inglewood Fault Zone and the axis of the Los Angeles Basin. The Santa Monica Mountains, located north of the site, are the southernmost of the east-west trending mountain ranges that comprise the Transverse Ranges Physiographic Province.

3.2 SITE AND SUBSURFACE CONDITIONS

The site of the proposed monopole is located on Shoreheights Drive, Malibu. The site of the proposed facility is relatively flat and covered with landscaping and concrete. The site is near the top of an approximately 2:1 (h:v), 100-foot high slope which descends in a westerly direction. The site drains in a westerly direction and is at an approximate AMSL of 450 feet. Existing underground utilities may be present within the site boundary.

The material encountered in the boring consisted of very dense silty sand with gravel to a depth of 15 feet and very dense clayey sand to the total depth drilled of 40 feet.

Groundwater was not encountered within the boring. Based on regional data, the historical highest groundwater level in the project vicinity is greater than 10 feet below the ground surface (CGS, 1997). The depth to groundwater may fluctuate, depending on rainfall and possible groundwater recharge or pumping activity in the site vicinity.

4.1 SEISMIC DESIGN

A seismic hazards screening was performed for this site to evaluate potential seismic hazards. The seismic hazards screening consisted of reviewing available data published by the California Geological Survey (CGS and, the 2007 California Building Code (CBC). The site is located in the United States Geological Survey Topanga Quadrangle. Data reviewed yielded the following Seismic Parameters:

Site Class	C
Spectral Response 'SMs'	1.850g
Spectral Response 'SM1'	0.859g
Fa	1.0
Fv	1.3

The computer program (EQFAULT, Version 3.00b) and data published by the CGS "The Revised 2002 California Probabilistic Seismic Hazard Maps," June 2003, were reviewed. Results of the fault search are presented in the Appendix. The search indicates that the Malibu Coast fault is 3.1 kilometers from the site.

The CGS (CDMG, 2000-003) does not delineate this site as being within an Alquist-Priolo Earthquake Fault Zone. However, with the active faults in the region, the site could be subjected to future strong ground shaking that may result from earthquakes on local to distant sources.

4.2 LIQUEFACTION POTENTIAL

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils exist below groundwater. The CGS has designated certain areas within southern California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site is not located within a zone that the CGS designates as a potential liquefaction hazard zone (CGS, 1997).

The boring indicates that the subsurface materials encountered at the project site generally consist of very dense granular material. Groundwater was not encountered within the boring, which was drilled to a maximum depth of 40 feet. The highest historical groundwater is greater than 10 feet beneath the ground surface (CGS, 1997). Based on this, we conclude that the potential for liquefaction at the site is low. Other geologic hazards related to liquefaction, such as lateral spreading, are therefore also low.

4.3 MONOPOLE FOUNDATION

4.3.1 Drilled Pier Foundation

The proposed monopole may be supported on a typical, large-diameter reinforced concrete drilled pier. However, drilling the pier may be difficult and require a special drill rig that is capable of penetrating the very dense material. The support from the pier will be derived from side friction for axial loads, and from passive soil resistance for lateral and over-turning forces.

Based on our exploratory boring, assumed design parameters are provided in the Load Data for Drilled Piers table in the attached Appendix. The allowable axial loads for the drilled pier are tabulated for various sizes of shafts. The allowable load is calculated for a safety factor of 3.0.

We understand that the pier will have a minimum diameter of 36 inches. We estimate that settlement of the pier would be less than ½ inch.

An equivalent fluid with a density of 300 pounds per cubic foot may be assumed for determining the lateral resistance of the soils against the projected width of the pier. The maximum lateral resistance should be capped at 3000 pounds per square foot at depths greater than 10 feet below the ground surface. The contribution of lateral resistance to a depth equal to two pier diameter or four feet, whichever is less, should be neglected.

The pier foundation should be designed and constructed in accordance with applicable procedures established by the Uniform Building Code (UBC) and the American Concrete Institute (ACI). The specifications should be patterned after recommendations included in the "Standards and Specifications for the Drilled Shaft Industry" published by the Association for Drilled Shaft Contractors (ADSC). We recommend that potential foundation contractors be pre-qualified with a heavy emphasis on local experience as recommended by ADSC.

Special drilling equipment may be required for excavating the pier shaft due to the very dense material. The contractor should be prepared to control possible caving. If temporary casing is used it should be removed as concrete is generally placed with at least a 3-foot head of concrete maintained within the casing to ensure the minimum required shaft diameter and prevent side wall collapse. Because the foundation design of the monopole counts on side friction and passive resistance for bearing capacity and lateral stability, casing must be removed. The use of temporary casing is at the discretion of the contractor. The pier shaft should not be left open for any prolonged period of time.

4.4 SHALLOW FOUNDATIONS

Shallow spread or continuous footings should have a minimum width of 18 inches and minimum embedment of 18 inches below lowest adjacent grade. In accordance with Section 4.7, "Site Preparation and Earthwork," any undocumented fill should be removed and replaced with compacted engineered fill. A representative of AESCO should confirm the depth of fill at the time of construction.

All foundations adjacent to any existing buildings, walkways and separately poured porches should be tied into the adjacent slabs to reduce separation and differential settlement. No. 5 bars by 30 inches long and spaced on 18-inch centers are recommended.

Footings should be founded on firm native soils or engineered fill.

Assuming these recommendations are followed, an allowable bearing pressure of 3000 psf may be assumed in the design of shallow spread foundations supported on engineered fill above firm native material. A passive soil resistance of 300 pcf and a coefficient of sliding resistance of 0.35 may be used for design against internal forces.

Under static loading, settlement of the footings designed according to our recommendations is estimated to be less than 1 inch. Differential settlement between similarly loaded footings is expected to be about one-half the total settlement.

4.5 GEOTECHNICAL RECOMMENDATIONS FOR VAULT DESIGN

Based on the results of our investigation, the proposed equipment vault may be supported on a conventional foundation system established in the underlying sandy material at the planned bottom of the vault. The exposed soil in the excavation for footings, vault base, or slab-on-grade should be swept clean of all loose material prior to structure placement. An allowable bearing pressure of 4200 psf may be assumed in the design of vault foundation supported on very dense clayey sand at a depth of approximately 17 feet.

A coefficient of sliding resistance of 0.35 may be used for evaluating resistance to sliding. An equivalent fluid density of 300 pcf may be used to calculate passive pressure assuming a level surface. Passive pressure should be reduced by one third if combined with sliding resistance.

Total lateral earth pressures acting on the wall during a seismic event will likely include the static force and the dynamic increment. Using the Mononobe-Okabe procedure, a dynamic lateral earth pressure increment (for a 0.48g) peak ground acceleration based on 10% probability of being exceeded in 50 years, (CGS) of 40H may be assumed for design purposes, where H (in units of feet) is the height of the soil behind the wall. This dynamic increment should be applied to the wall as a triangle pressure over the wall height starting from the bottom of the wall to the top, and are added to the static earth pressures. The lateral earth pressures recommended above are based upon the assumption that the backfill is granular, the ground surface behind the wall is level, and the wall backfill is well drained. The pressure should be increased by 35 percent for sloping backfill with a 2:1 (H:V) slope.

4.5.1 Lateral Earth Pressures

Walls below grade will be subjected to lateral earth pressures from the retained soils and surcharge loads. Accordingly, these structures should be designed to resist appropriate lateral earth pressures.

For design purposes, a triangular distribution of lateral earth pressures with an equivalent fluid pressure of 29 pounds per cubic foot (pcf) should be used in design of walls below grade for a restrained condition. This assumes a horizontal grade behind the wall.

The design values assume free-draining backfill materials are placed behind the wall. Surcharge pressures (dead or live) should be added to the above lateral earth pressures where surcharge loads may be located adjacent to the wall. Surcharge pressures should be applied as a uniform (rectangular) pressure distribution by using a pressure equal to 0.5 times the surcharge pressures. Vertical surcharges set back behind the wall a horizontal distance greater than the wall height need not be added to the design pressure. The above coefficients assume a uniform surcharge load.

4.5.2 Wall Backfill

Backfill behind walls below grade should consist of granular backfill that is placed directly above and behind the drain material. To reduce the potential for settlement of backfill, it is essential that wall backfill be properly compacted in lifts. The minimum compaction standard for wall backfill should be 90 percent relative compaction. In the event that the wall backfill will support structures or facilities, the compaction standard should be increased to 95 percent relative compaction. Heavy compaction equipment should not be used within 5 feet of the wall. Small hand-operated compaction equipment should be used adjacent to the wall so as not to overstress the wall. The lift thickness with the smaller equipment should not be more than six inches.

4.6 EXCAVATION AND SHORING

We anticipate that installation of the vault will generally involve excavating up to nine feet below grade in clayey material. The material can be classified as soil type (B) based on CAL-OSHA classification. Temporary construction slopes should not be steeper than 1:1 (H:V). Alternatively, shoring may be used to support the excavation. For the proposed vault excavation, shoring may consist of soldier piles and lagging or another suitable system to retain the sides of the excavation. Shoring should be designed by a licensed engineer experienced in shoring design and submitted for our review.

For the design of cantilever, a minimum equivalent fluid pressure of 35H psf per foot of depth below grade may be used, where (H) is the height in feet. For the design of braced shoring supporting a sloping grade, we recommend such shoring be designed using a rectangular-shaped distribution of lateral earth pressure for a maximum earth pressure of 30H (in psf).

For the design of soldier piles spaced at least three diameters on centers, the passive resistance of the soils adjacent to the piles may be assumed to be 300 psf per foot of embedment depth for the projected width of the pile, up to 3000 psf maximum. The soldier piles may be installed in drilled excavations. Soldier pile members placed in drilled holes should be properly backfilled with sand/cement slurry or lean concrete in order to develop the required passive resistance.

The design of the shored excavation should be performed by an engineer knowledgeable and experienced with the on-site soil conditions. The contractor should be aware that slope height, slope inclination or excavation depths should in no case exceed those specified in local, state or federal safety regulations, e.g. OSHA Health and Safety Standards for Excavation, 29 CFR Part 1926, or successor regulations. Such regulations are strictly enforced and, if not followed, the owner or the contractor could be liable for substantial penalties.

4.7 SITE PREPARATION AND EARTHWORK

All grading and site preparation should be observed by experienced personnel reporting to the project Geotechnical Engineer. Our field monitoring services are an essential continuation of our studies to confirm and correlate the findings and our prior recommendations with the actual subsurface conditions exposed during construction, and to confirm that suitable fill soils are placed and properly compacted.

Excavation side slopes should be cut at a gradient no steeper than 1:1 (horizontal to vertical), and excavations should not extend below an imaginary 1:1 inclined plane projecting below the bottom edge of adjacent existing foundations and/or utilities unless properly shored or specifically analyzed further. All excavations should be observed by AESCO to confirm that all unsuitable material is removed from beneath the planned construction prior to placing fill if disturbed, the exposed excavation bottom should be scarified and compacted prior to fill placement.

Excavations below the final grade level should be properly backfilled using approved fill material. The backfill and any additional fill should be placed in loose lifts less than 8 inches thick, moisture conditioned to 0 to 4 percent above optimum water content, and compacted to a minimum of 90 percent relative compaction as determined by ASTM Test Method D1557. When engineered fill underlies structural elements such as slabs or footings, it should be compacted to at least 95 percent relative compaction. Engineered fill should consist of soils with a maximum particle size of 3 inches, at least 80 percent passing the ¾-inch sieve and with an expansion index not greater than 20. Fill materials should be free of construction debris, roots, organic matter, rubble, contaminated soils, and any other unsuitable or deleterious material as determined by the Geotechnical Engineer. The on-site soils may be suitable for use as compacted fill, provided the soil is free of any deleterious materials and satisfies the expansion index criteria. We recommend that if imported fill material is used, it be reviewed for acceptability by the Geotechnical Engineer prior to importing it to the site for use as engineered fill.

A representative of the geotechnical engineer should observe all footing and slab subgrade surfaces and confirm that the exposed materials are firm. If loose, spongy, soft or other unacceptable materials, including undocumented fill, are encountered in the subgrade they should be removed to firm materials as determined by the geotechnical engineer's representative and replaced with either concrete or compacted engineered fill.

4.8 SOIL CORROSIVITY

The results of pH, soluble chloride, and soluble sulfate laboratory tests on a sample of the near surface soils are summarized in the following table:

Soil Test	Test Results	Corrosion Potential
Soluble Sulfates (per CA 417)	21 ppm	Negligible sulfate attack on concrete.
Soluble Chlorides (per CA 422)	60 ppm	Moderate corrosive potential to buried ferrous metals
PH	7.6	Mild to moderate corrosion potential to buried ferrous metals

Concrete should be designed in accordance with the 2007 CBC, ACI 318 Section 4.3, Table 4.3.1 (2005). As the potential for sulfate attack on concrete appears negligible Type II Portland cement may be used with no water to cement ratio for the purpose of sulfate attack abatement. All subgrade soils should be moistened to 125% of optimum moisture prior to the concrete pour. The minimum compressive strength of concrete shall be 3000 psi at 28 days and maximum slump during placement shall be five inches. A qualified inspector, under the supervision of a professional engineer, shall inspect the concrete placement.

The test results generally indicate that the on site soils can be classified as moderate corrosive potential to buried metallic structures (e.g. pipes). As a minimum, buried metal piping should be protected with suitable coatings, wrappings, or seals. As an alternative, utility piping may be buried in PVC lined trenches and backfilled with clean sand. The width of the trenches should be a minimum of three times the diameter of the pipes. A corrosion consultant should be retained if a more detailed evaluation or a protection system is desired. AESCO recommends that additional corrosivity evaluation shall be performed during grading operations and for any imported fill to ensure that corrosivity characteristics have not changed.

4.9 UTILITY TRENCHES

It is anticipated that the on-site soils will provide suitable support for underground utilities and piping that may be installed. Any soft and/or unsuitable material encountered at the bottom of excavations for such facilities should be removed and be replaced with an adequate bedding material. A non-expansive granular material with a sand equivalent greater than 30 should be used for bedding and shading of utilities.

The on-site soil may be used for backfill of utility and pipe trenches from one foot above the top of the pipe to the final ground surface, provided the material is free of organic matter and deleterious substances. Trench backfill should be mechanically placed and compacted in 8-inch lifts to at least 90 percent of the maximum dry density as determined by ASTM Test Method D 1557 (i.e. 90 percent relative compaction). Where trenches are placed beneath slabs or footings the backfill shall satisfy the gradation and expansion index requirements of engineered fill (see Section 4.7). Flooding or jetting for placement and compaction of backfill is not recommended.

4.10 SECTION 111 STATEMENT

Based on the findings summarized in this report, it is our professional opinion that the proposed construction will not be subject to a hazard from settlement, slippage, or landslide, provided the recommendations of this report are incorporated into the proposed construction. It is also our opinion that the proposed construction will not adversely affect the geologic stability of the site or adjacent properties provided the recommendations contained in this report are incorporated into the proposed construction.

4.11 CONSTRUCTION OBSERVATIONS AND FIELD TESTING

As geotechnical engineer of record, construction observation and field testing services are an essential continuation of this geotechnical study to confirm and correlate our findings and recommendations with the actual subsurface conditions exposed during construction. As such, to maintain the status of geotechnical engineer of record, AESCO should be present to observe and provide testing during the following construction activities:

- Observations of drilled pier
- Excavation and backfill for footings and subgrade for any slabs on grade
- Placement of all fill and backfill
- Backfilling of utility trenches
- Installation of concrete and rebar

5.1 LIMITATIONS

It must be recognized that conclusions reached in this report are based on conditions, which exist at the boring location and are assumed to exist over the entire site. In any subsoil investigation, it is necessary to assume that the subsoil conditions between boring(s) do not change significantly. The number of the borings, locations, and spacing are chosen in such a manner as to decrease the possibility of undiscovered anomalies, while considering the nature of loading, size, existing structures, and cost of the project. Note that the boring(s) were placed as close to the location of the proposed structure(s) as possible. Consequently, careful observations must be made during construction to detect significant deviations of actual conditions throughout the construction area from those inferred from the exploratory borings.

In the event that significant changes in design loads or structural characteristics are made, AESCO should be retained to review our original design recommendations and their applicability to the revised design plans. In this way, any required supplemental recommendations can be made in a timely manner.

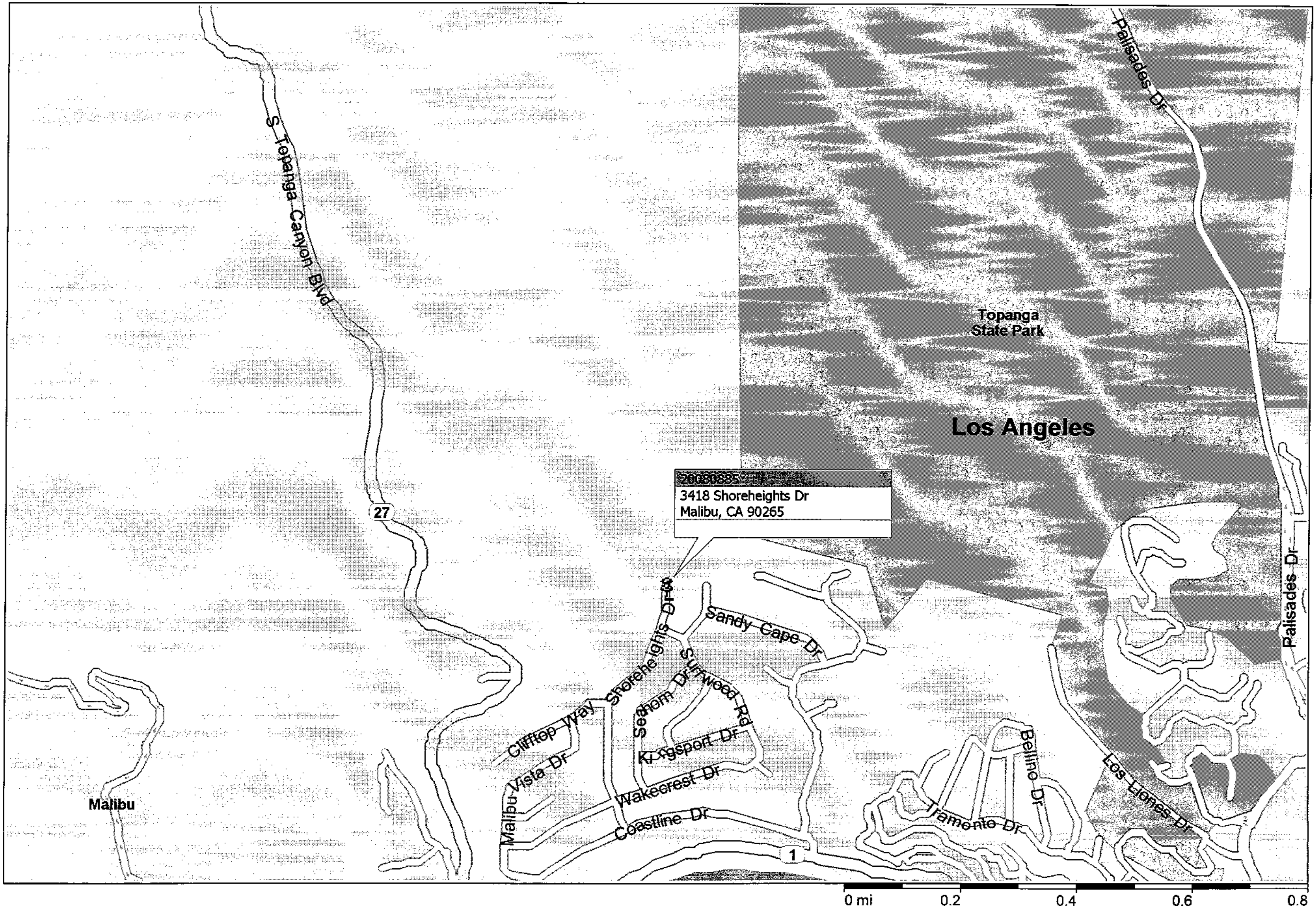
Should any unusual conditions be encountered during construction, this office should be notified immediately so that further investigations and supplemental recommendations can be made. Geotechnical observations and testing should be provided on a continuous basis during grading, excavation, and installation of the foundations. If parties other than AESCO are engaged to provide geotechnical services during construction they will be required to assume the full responsibility for the geotechnical phase of the project by adhering to the recommendations of this report.

Analysis by:

Adam Chamaa, P.E., G.E.

APPENDIX
SITE PLAN

20080885



S Topanga Canyon Blvd
27
Palisades Dr
Topanga
State Park
Los Angeles
20080885
3418 Shoreheights Dr
Malibu, CA 90265
Sandy Cape Dr
Shoreheights Dr
Surfwood Rd
Seahorn Dr
Kingsport Dr
Clifftop Way
Vista Dr
Malibu
Wakecrest Dr
Coastline Dr
1
Bellino Dr
Tramonto Dr
Los Liones Dr
Malibu
0 mi
0.2
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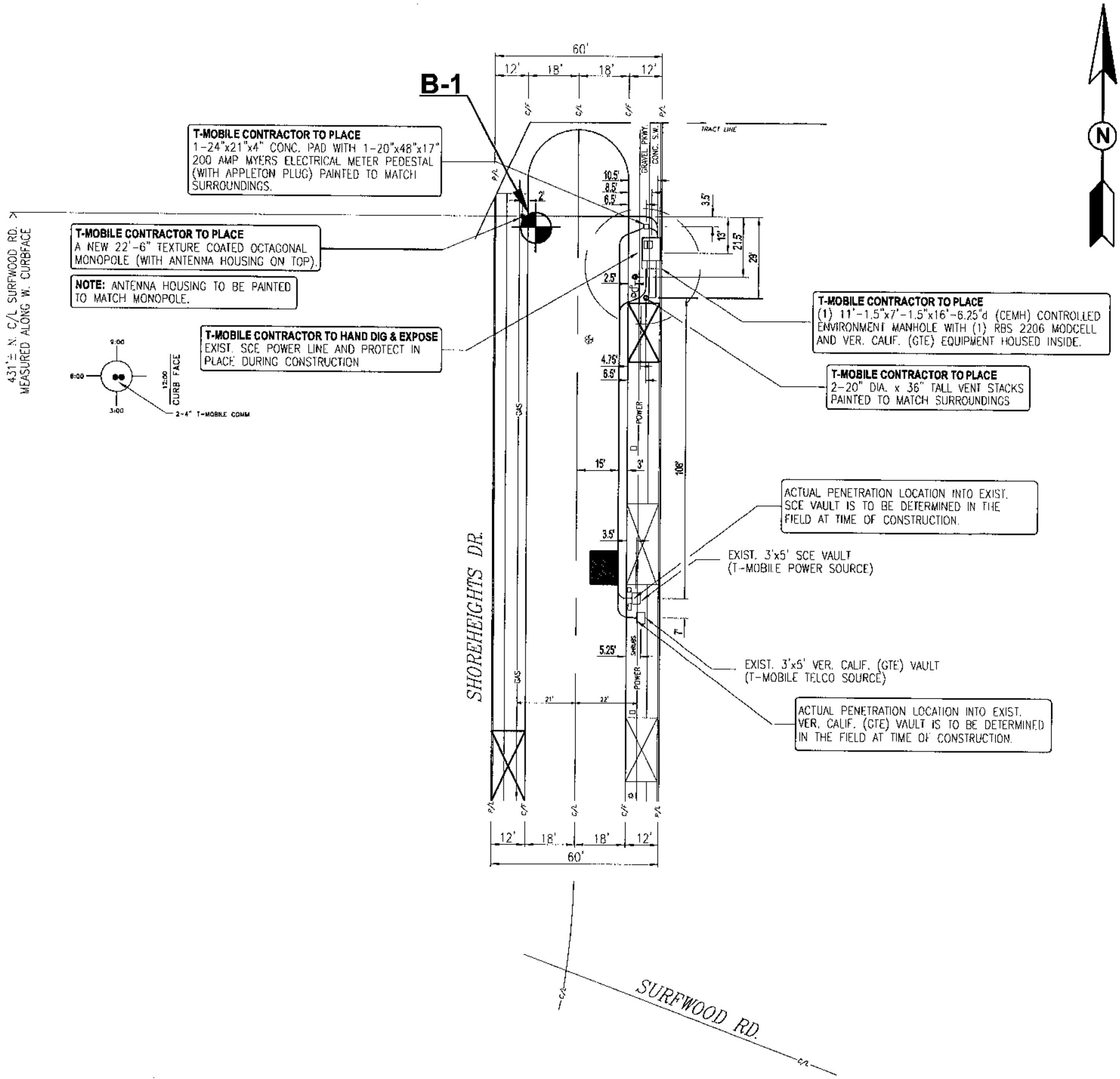


LEGEND

B-1 Approximate Location of Boring

T-Mobile	
Project No. : 20080885-A3309	Scale: *1 inch = ±50 feet*
Site Name: SHOREHEIGHTS DRIVE R.O.W.	Site Number: SV11110D
Site Address: 3418½ SHOREHEIGHTS DRIVE, MALIBU, CA 90265	
SITE PLAN	Date: 07-29-08 / Figure 1

AESCO
Technologies, Inc.

APPENDIX
LOG OF BORING B-1

MAJOR DIVISION			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVEL (LITTLE OR NO FINES)		GW	WELL GRADED GRAVELS, GRAVEL SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY GRADED GRAVELS, GRAVEL SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVEL WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL SAND SILT MIXTURE
				GC	CLAYEY GRAVELS, GRAVEL SAND CLAY MIXTURES
MORE THAN 50% BY WEIGHT OF MATERIAL IS LARGER THAN 200 SIEVE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINE (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT <50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% BY WEIGHT OF MATERIAL IS SMALLER THAN 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT >50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLATICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

AESCO Technologies, Inc.

UNIFIED SOIL ~~CLASSIFICATION~~ SYSTEM

KEY		
	☒ Split Spoon Sample (SPT)	▼ Ground Water Level
	[C] California Modified Sample	N SPT Blows/ft
	◫ Hand Auger Sample	P Penetrometer TSF

LOG OF BORING NO. B - 1

AESCO TECHNOLOGIES, INC.

Project: SV11110D Shoreheights Drive ROW

Location: 3418 1/2 Shoreheights Drive Malibu, CA

WATER: Not Encountered

Client: T-Mobile

Date: 07/10/08

Logger:

Project No. 20080885-A3309

DRILLING: Hollow Stem Auger

FIELD DATA: SOIL SYMBOL	DEPTH (FT)	TESTS: N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	DESCRIPTION OF STRATUM
SM			1.8											Light brown silty SAND (SM), dry, w/gravel
	3													
	5	N=50/4"	2.9											Very dense at 3'
C	7	N=50/5"	4.1	104.9						27.5	0	30		Brown at 5'
	8													
	10	N=50/5"	5.0											Moist at 8'
	13													
C	15	N=50/6"	5.3	73.4										
SC	18													Brown clayey SAND (SC), very dense moist
	20	N=50/4"	5.3											
	23													
C	25	N=50/3"	4.5	97.9						44.5	0	22		
	28													
	30	N=50/3"	5.5											
	33													
C	35	N=50/3"		94.1										
	38													
	40	N=50/6"												No Sample Recovery at 38'

Boring Terminated at 40 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
SM
SC

Ground Water Level
Hydrostatic Ground Water Level

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

APPENDIX
LABORATORY TEST DATA

Aesco Technologies, Inc.
17782 Georgetown Lane
Huntington Beach, California 92647

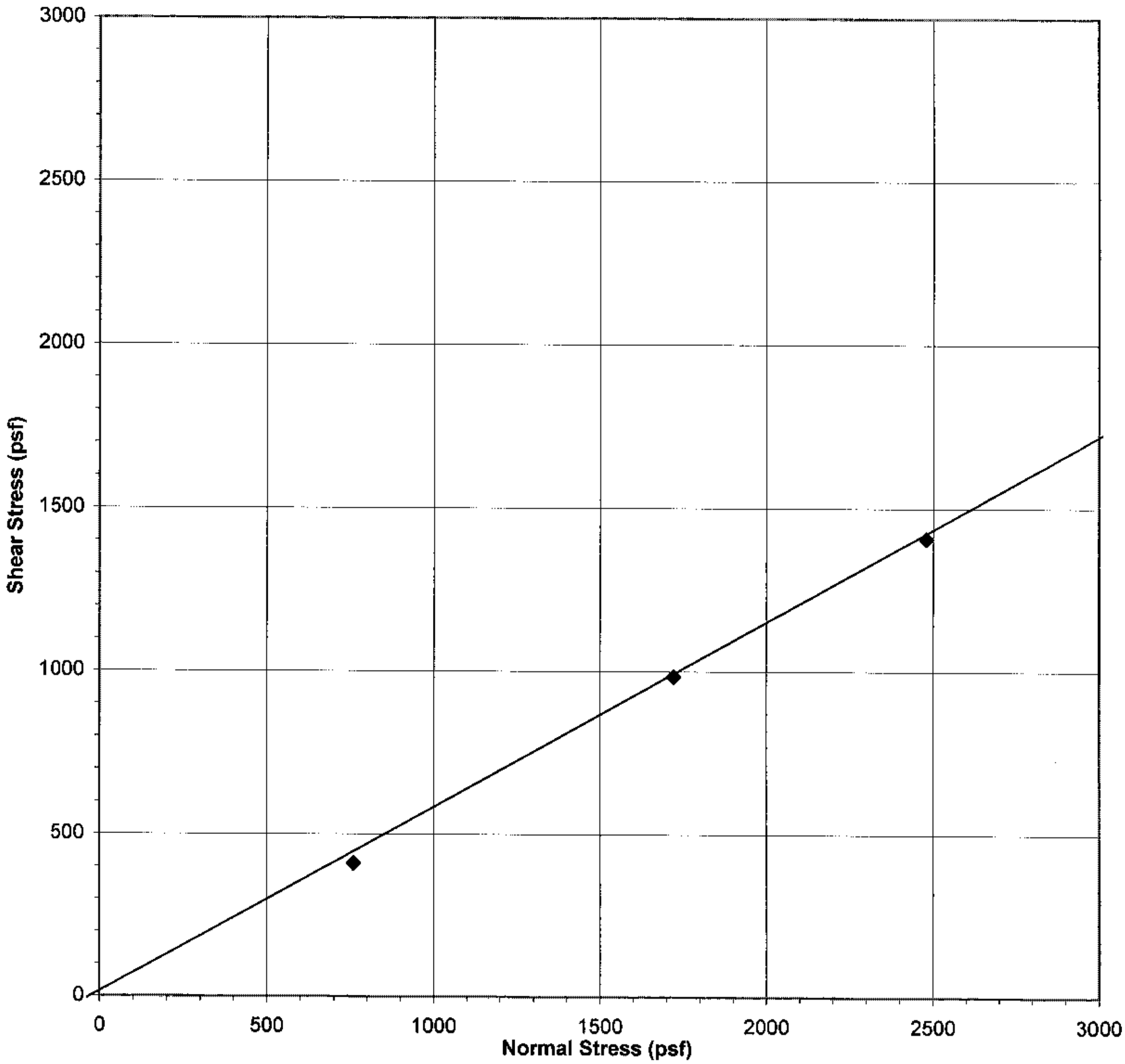
SITE/CLIENT: SV11110D

Project NO: 20080885-A3309 BORING NO: B-1 DEPTH: 5-7

W= 4.1% γd= 94.7 PCF C=0 PSF ϕ= 30 deg

UNDISTURBED: * REMOLDED: RESIDUAL:

CLASSIFICATION: SM



Aesco Technologies, Inc.
17782 Georgetown Lane
Huntington Beach, California 92647

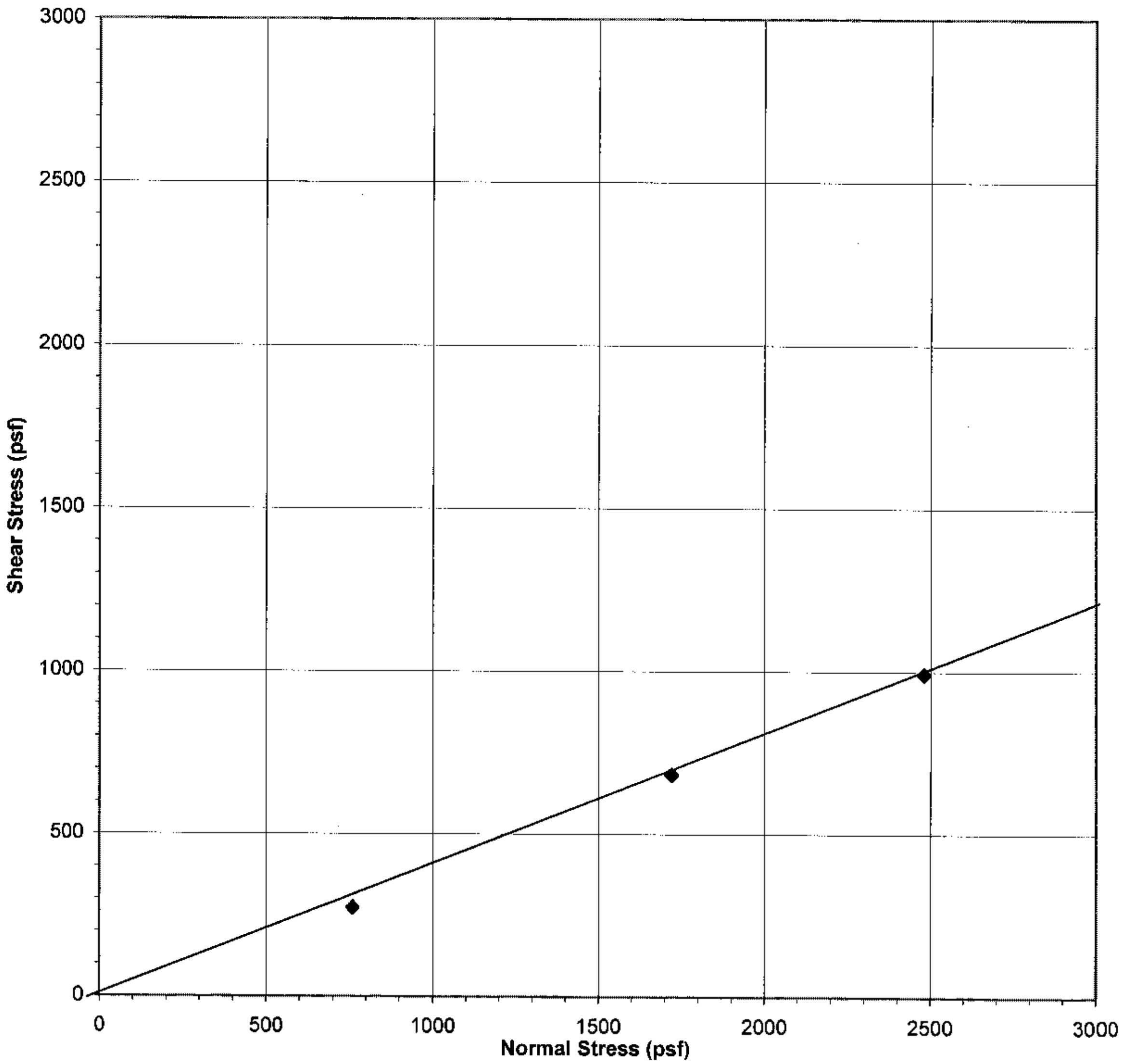
SITE/CLIENT: SV11110D

Project NO: 20080885-A3309 BORING NO: B-1 DEPTH: 23-25

W= 4.5% γd= 98.0 PCF C=0 PSF ϕ= 22 deg

UNDISTURBED: * REMOLDED: RESIDUAL:

CLASSIFICATION: SC



APPENDIX
SEISMIC DESIGN

Project Name = SV11110D
Conterminous 48 States
2006 International Building Code
Latitude = 34.0487
Longitude = -118.5723
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.01 deg grid spacing

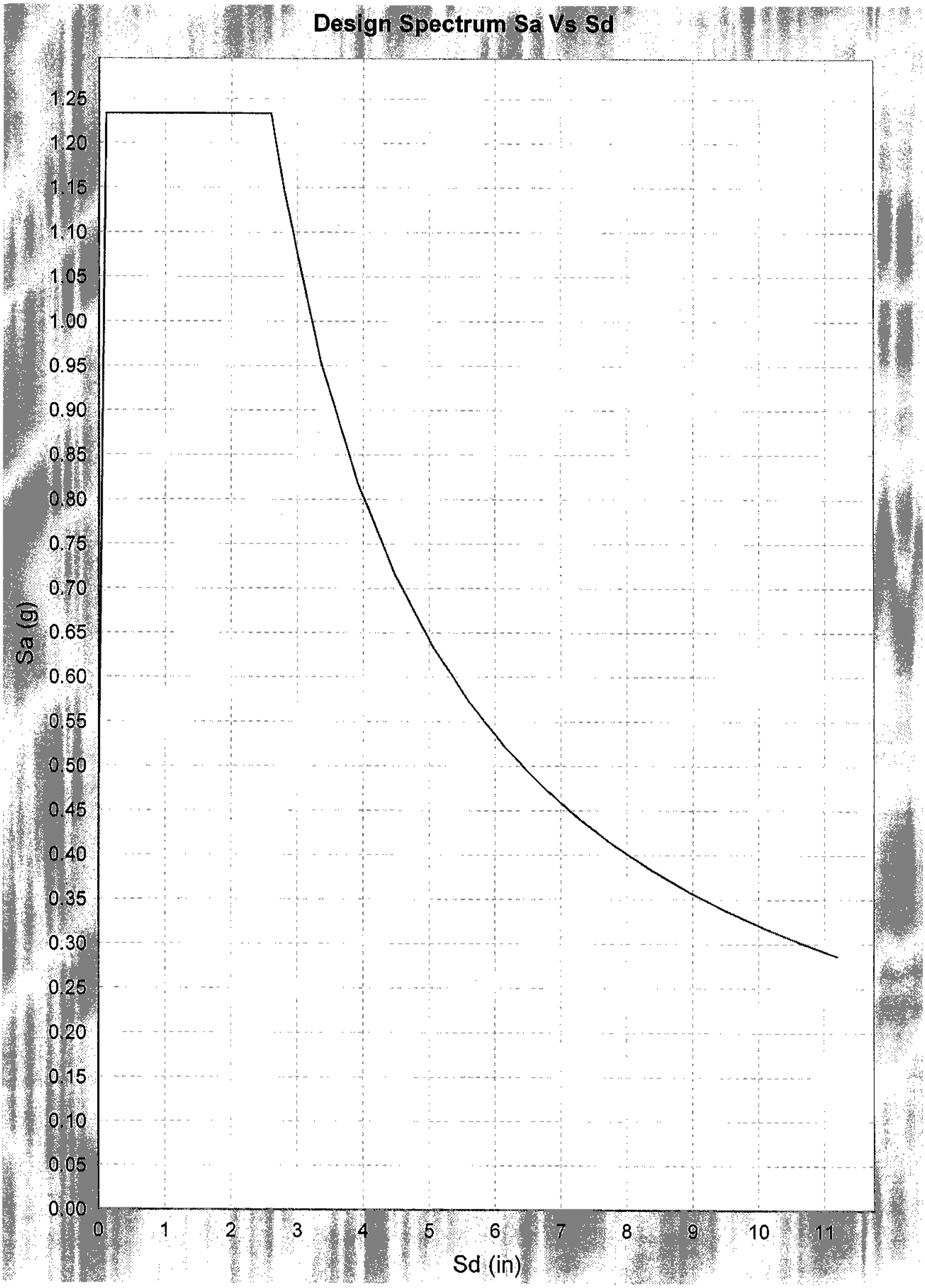
Period (sec)	Sa (g)
0.2	1.850 (Ss, Site Class B)
1.0	0.661 (S1, Site Class B)

Conterminous 48 States
2006 International Building Code
Latitude = 34.0487
Longitude = -118.5723
Spectral Response Accelerations SMs and SM1
SMs = FaSs and SM1 = FvS1
Site Class C - Fa = 1.0 ,Fv = 1.3

Period (sec)	Sa (g)
0.2	1.850 (SMs, Site Class C)
1.0	0.859 (SM1, Site Class C)

Conterminous 48 States
2006 International Building Code
Latitude = 34.0487
Longitude = -118.5723
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class C - Fa = 1.0 ,Fv = 1.3

Period (sec)	Sa (g)
0.2	1.234 (SDs, Site Class C)
1.0	0.573 (SD1, Site Class C)


Design Spectrum Sa Vs Sd
Sa (g)
1.25
1.20
1.15
1.10
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
1
2
3
4
5
6
7
8
9
10
11
Sd (in)

```
***********************
*                     *
*    E Q F A U L T    *
*                     *
*    Version 3.00     *
*                     *
***********************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 20080885-A3309

DATE: 08-01-2008

JOB NAME: SV11110D

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 34.0487
   SITE LONGITUDE: 118.5723

SEARCH RADIUS: 62.1 mi

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
MALIBU COAST	1.9(3.1)	6.7	1.016	XI
SANTA MONICA	2.5(4.1)	6.6	0.947	XI
PALOS VERDES	5.7(9.2)	7.1	0.610	X
ANACAPA-DUME	8.3(13.4)	7.3	0.632	X
HOLLYWOOD	9.8(15.8)	6.4	0.369	IX
NEWPORT-INGLEWOOD (L.A.Basin)	11.6(18.7)	6.9	0.356	IX
NORTHRIDGE (E. Oak Ridge)	14.7(23.6)	6.9	0.318	IX
COMPTON THRUST	14.9(24.0)	6.8	0.296	IX
VERDUGO	18.8(30.3)	6.7	0.218	IX
SANTA SUSANA	19.3(31.0)	6.6	0.200	VIII
SIERRA MADRE (San Fernando)	20.3(32.6)	6.7	0.201	VIII
RAYMOND	20.8(33.5)	6.5	0.172	VIII
ELYSIAN PARK THRUST	20.8(33.5)	6.7	0.195	VIII
SIMI-SANTA ROSA	21.8(35.1)	6.7	0.185	VIII
HOLSER	23.4(37.7)	6.5	0.150	VIII
SIERRA MADRE	23.7(38.1)	7.0	0.203	VIII
OAK RIDGE (Onshore)	23.9(38.4)	6.9	0.190	VIII
SAN GABRIEL	23.9(38.5)	7.0	0.193	VIII
SAN CAYETANO	29.8(48.0)	6.8	0.138	VIII
WHITTIER	32.1(51.6)	6.8	0.123	VII
CLAMSHELL-SAWPIT	33.1(53.2)	6.5	0.097	VII
OAK RIDGE(Blind Thrust Offshore)	38.4(61.8)	6.9	0.108	VII
VENTURA - PITAS POINT	38.8(62.5)	6.8	0.098	VII
SAN JOSE	39.3(63.3)	6.5	0.076	VII
CHANNEL IS. THRUST (Eastern)	40.0(64.3)	7.4	0.144	VIII
SANTA YNEZ (East)	41.1(66.2)	7.0	0.108	VII
SAN ANDREAS - Mojave	42.6(68.5)	7.1	0.112	VII
SAN ANDREAS - 1857 Rupture	42.6(68.5)	7.8	0.188	VIII
MONTALVO-OAK RIDGE TREND	42.8(68.8)	6.6	0.073	VII
CHINO-CENTRAL AVE. (Elsinore)	44.9(72.2)	6.7	0.074	VII
SAN ANDREAS - Carrizo	45.2(72.8)	7.2	0.113	VII
M.RIDGE-ARROYO PARIDA-SANTA ANA	45.7(73.6)	6.7	0.072	VII
CUCAMONGA	47.3(76.1)	7.0	0.087	VII
RED MOUNTAIN	48.4(77.9)	6.8	0.072	VII
NEWPORT-INGLEWOOD (Offshore)	49.2(79.2)	6.9	0.078	VII
SANTA CRUZ ISLAND	54.2(87.2)	6.8	0.062	VI
ELSINORE-GLEN IVY	55.2(88.9)	6.8	0.062	VI
GARLOCK (West)	57.5(92.5)	7.1	0.076	VII
PLEITO THRUST	58.0(93.4)	7.2	0.077	VII
BIG PINE	58.1(93.5)	6.7	0.053	VI

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT		
		MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.

****

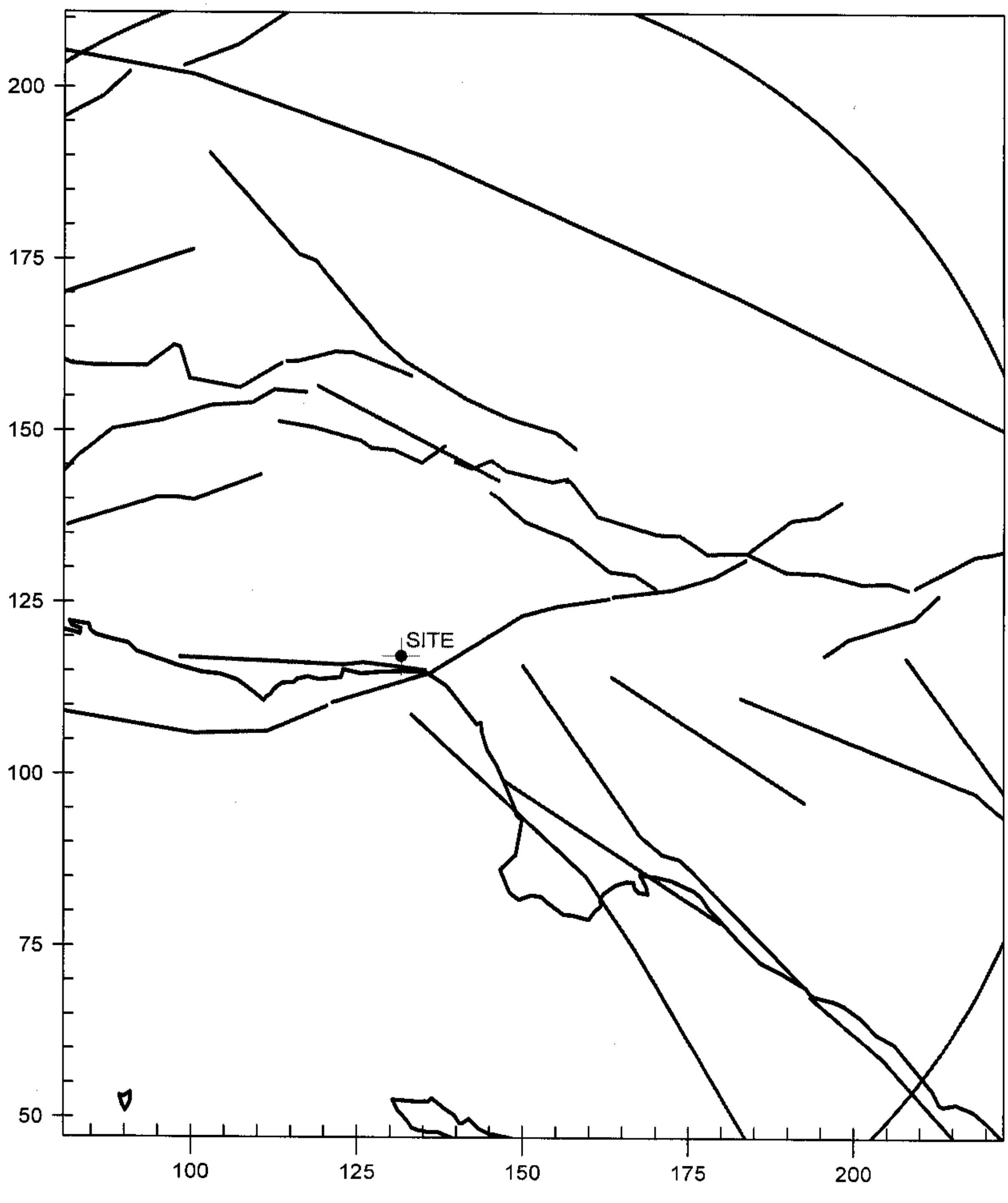
-END OF SEARCH- 40 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE MALIBU COAST FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.9 MILES (3.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 1.0159 g

CALIFORNIA FAULT MAP

SV11110D



APPENDIX
LOAD DATA FOR DRILLED PIERS

AESCO TECHNOLOGIES
18548 Beach Blvd.
Huntington Beach, California 92648

(714) 593-2567
Fax (714) 593-0022

04 Aug 08
20080885-A3309

SV11110D
Shoreheights Drive ROW

ALLOWABLE LOADS (KIPS)
FOR DRILLED SHAFTS (skin friction only)
FACTOR OF SAFETY = 3.0

TIP DEPTH BELOW SURFACE	SHAFT DIAMETER (IN.)							
(FT.)	24	30	36	42	48	54	60	66
0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0
7	1	2	2	2	2	3	3	3
10	3	4	5	6	6	7	8	9
15	8	10	12	14	16	18	20	22
20	14	17	21	24	28	31	35	38
25	22	27	32	38	43	48	54	59
30	31	39	46	54	62	70	77	85
35	42	52	63	73	84	94	105	115
40	55	68	82	96	110	123	137	151